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SELECTION OF UPLIFT COEFFICIENTS FOR UPLIFT DESIGN OF TRANSMISSION TOWER FOUNDATIONS

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ABSTRACT

This report presents and discusses enhancements to the Ku theory for uplift capacity of augered pier foundations in granular soils. Ku-theory was proposed by Meyerhof and Adams in 1968 and was supported by a limited range of field and laboratory data. In this paper a thorough review of the uplift load test data now available in the literature is made. The data is analyzed by the Ku-theory to yield correlations between Ku and depth to diameter ratio of foundations. Gaps in the data are supplemented by further full scale tests and laboratory model tests. Statistical parameters for Ku have been derived from this data. A probability based method for uplift design of augered footings is proposed, incorporating these statistical Ku parameters.

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SELECTION OF UPLIFT COEFFICIENTS FOR UPLIFT DESIGN OF TRANSMISSION TOWER FOUNDATIONS

INTRODUCTION

Within Ontario Hydro, cylindrical augered footings are the preferred method of support for lattice type transmission towers. Due to horizontal wind and ice loading acting on the conductors, these footings experience both compression and uplift forces. Resistance to uplift forces normally governs the design and the design is largely based on ultimate capacity. In cohesionless soils, the Ku theory developed by Meyerhof and Adams (1968) forms the basis for uplift design within Ontario Hydro (Adams, 1975). Using this theory ultimate uplift capacity (Qu) is calculated as follows:

$$Q_u = 1/2 \gamma B \pi D^2 K_u \tan \phi + W \dots\dots\dots(1)$$

where γ is the soil density
B is the footing diameter
D is the footing depth
 ϕ is the angle of internal friction of
the soil
W is the weight of the footing
and Ku is the uplift (or lateral earth pressure) coefficient.

This theory was initially developed for buried plates and was extended to cylindrical footings by regarding Ku as an empirical uplift coefficient (Meyerhof and Adams 1968). The Ku factor was found to be dependent on both soil friction (ϕ) and the depth to diameter ratio (D/B) of the footing.

Figure 1 shows a family of bilinear K_u versus D/B curves for different ϕ values. The knee in the bilinear curves stems from the critical depth effect. For footings buried below a certain critical depth, no increase in K_u is obtained with increasing footing depth. The value of K_u is solely dependent on soil strength. For footings shallower than the critical depth, K_u increases with increasing footing depth and the slope of the relationship is dependent on soil strength.

Early model tests on cylindrical footings provided support for the K_u theory. Field scale tests at Ontario Hydro showed reasonable agreement (Adams and Radhakrishna 1977) with the model test data (Figure 2) and with the theory (Figure 1). However, the field scale tests were virtually all done on footings having D/B values less than 4 so scarcely any field data was available to confirm the theory at higher D/B ratios or at K_u values much above 2. Apparently as a result, Ontario Hydro's design guide (Adams, 1975) did not recommend K_u values larger than 2. While this may have been appropriate for the then common 230 kV tower line with footings having D/B ratios on the order of 4 or less, it may be unduly penalizing today's 500 kV tower line footings with D/B ratios ranging up to 10 or more. Confirmation of the K_u theory at large D/B ratios is now required.

Attempts have been made to theoretically estimate the limits for K_u in terms of earth pressure theories and yield theories for cohesionless soils (Kulhawy et al 1979, Kulhawy et al 1982, Chaudhuri and Symons 1983). However, to date, estimation of K_u has been largely based on empirical correlations between K_u , depth to width ratios of footing and internal friction angle. Therefore, it was considered necessary to build up a large data base from field and laboratory tests, statistically analyze it and examine the validity of the K_u vs D/B correlations proposed by Adams and Hayes (1967) over a range of D/B of practical interest to designers.

DATA BASE

A review of the literature located uplift test information for 146 cylindrical footings. Of these footings, 78 were "full scale" field tests and 68 were laboratory model tests. Sixteen additional full scale tests (Cragg 1985) and 24 additional model tests (Cragg and Tsang, 1985) were performed to supplement the literature data. The additional testing strengthened the data base in the dense range for footings having D/B values from 3 to 7. All the foundation test data collected for this study is summarized, by scale of testing and by soil density, in Tables I to VIII. Details of the case studies referred to in these tables are contained in Appendix A. In the majority of cases, K_u was backcalculated from the maximum foundation load derived from the literature load curves. It should be noted that the information on soil strength (ϕ) in the literature and how it was obtained was generally not well

documented, forcing the writers to assume values in some instances, as shown in the tables. After deleting the test points judged to be unreliable (see Table IX), 160 foundation tests remained for analysis. Table X shows the distribution of this data by soil density and by scale of test (ie field "full scale" or laboratory "model"). The data is reasonably distributed by soil density and scale of footing tests, even though 2/3 of the data was supplied by four authors and 1/3 originated from Ontario Hydro in particular (Table XI). To avoid biasing the data towards laboratory testing or Ontario Hydro work recent work by Radhakrishna and Tsang (1986) was not used in the analysis.

DATA ANALYSIS

Each group of data - dense, compact and loose - has been subjected to statistical analysis. This analysis included (a) linear regression of (Ku, D/B) data with D/B values less than the knee of the bilinear "curves" and determination of mean Ku values beyond the knee point, (b) determination of standard errors and standard deviations for the preceding analysis, and (c) higher order (non-linear) regressions of (Ku, D/B) data.

A comparison between the total laboratory and the total field data in each density category has also been done in this manner.

Linear regressions of laboratory plus field data for each density classification are presented in Figures 3, 4 and 5. Also shown in these figures is a bandwidth of one standard error for the sloped regression line and one standard deviation for the horizontal average value line. Mathematically, the standard error and standard deviation are almost identical. Roughly 2/3 of the Ku values fall within this bandwidth. Similar regressions were performed separately for field and laboratory data. While these regressions are not shown, the statistical parameters obtained are compared with those for the combined data in Table XII. It may be seen from Table XII that the standard errors, standard deviations and average Ku values for the three data sets are quite similar. No data set is unduly influencing the overall correlation except where it dominates the number of observations (n). Table XII shows this domination in number of data points occurs in two places. Dense soils with D/B greater than 6.5 are dominated by model test data. Compact soils with D/B less than 5.0 are dominated by field test data.

Higher order regressions of Ku and D/B plot almost on top of the bilinear relationships shown in Figures 3 to 5. Use of these second and third order curve fits does not appreciably improve the correlation coefficients (r) shown in Table XII. The relationships shown in Figures 3 to 5 were combined to form the overall graph shown in Figure 6.

The soil strengths reported for the data in each density band are:

Soil Density	$\bar{\phi}$ (mean value)	σ_{ϕ} (standard deviation)
Dense	39 ⁰	+/-3 ⁰
Compact	33 ⁰	+/-3 ⁰
Loose	30 ⁰	+/-2 ⁰

Applying these soil strengths to Figure 6 results in some overlap of values in the compact and loose bands. A number of laboratory tests in the loose range had ϕ values exceeding 30⁰. Examination of the data showed that Ku for loose laboratory tests was much lower than for comparable field tests. A lower laboratory Ku would be obtained if ϕ had been valued too high. As the field test values are representative of soils with ϕ values of 29⁰ $\pm$ 1⁰ and the laboratory test ϕ values appear to have been overestimated, the loose range of Ku depicted on Figure 6 is believed to represent soils with ϕ values of 29⁰ $\pm$ 1⁰. The combined correlation plot treated the minor band overlaps of the various soil densities in a conservative manner.

COMPARISON TO ORIGINAL WORK

In Figures 3, 4 and 5 the data for dense, compact and loose soils, respectively, is compared to the laboratory work of Adams and Hayes 1967 (shown in Figure 2) for D/B values less than twelve. The compiled data points generally compare well with this early laboratory model work. The new data from the recent Ontario Hydro field tests at Essa (designated E on Figure 3) fall along the lower boundary of the laboratory defined limits for the dense soil. This is reasonable because the Essa soil density is borderline between compact and dense. The data for compact sand spans the gap between the loose and dense bands defined by the laboratory work (Figure 4). In loose soils, some data points at low D/B values plot above the expected range but the other values compare well with the laboratory work (Figure 5).

In comparing the data points to the theoretical boundaries, the following trends were observed. The dense data generally plots above the $\phi = 36^0$ line, taken to be the compact to dense boundary (Figure 6). As data in the very dense range is very rare no attempt has been made to separate the dense and very dense ranges. In the compact range, $\phi = 30^0$ to 36⁰, there is considerable scatter of the data outside of the theoretical boundaries. For soils in the loose range, most points fall below the theoretical loose line for $\phi = 30^0$.

Comparing the statistically derived bandwidths with the theoretical values (Figure 6) shows reasonable agreement in the dense soil range (ϕ from 36° to 45°). The statistical bandwidth is wider in the compact soil range (ϕ from 30° to 36°) than anticipated by the theory. In the loose soil range (ϕ less than 30°), the disparity between theory and observation becomes larger. K_u is much lower in loose soils than expected by theory. This divergence from theory with progressively looser soils is believed to be due to construction effects. Looser soils are more susceptible to construction disturbance and weakening than dense soils. The lower bound of the loose range has a K_u value of about 0.5 which is approximately the value of the "at rest" earth pressure coefficient K_0 . Poorooshasb (1980) has shown that K_0 is the practical lower bound for lateral earth pressures acting on footings subjected to uplift. Tsang (1987) has shown that K_0 is the most reliable estimator of uplift capacity in loose sands.

On a more general basis, the statistical data supports the concepts on which the theory is based:

- K_u is increasingly dependent on soil density (and ϕ value) as density (and ϕ value) increase
- beyond a critical D/B ratio, K_u is independent of D/B and entirely a function of ϕ .
- at low ϕ values K_u depends on ϕ alone and not on D/B.

These observations show the strong dependence of K_u on ϕ , particularly at higher ϕ values. Accurate measurement of ϕ in denser soils is therefore of critical importance in uplift design. The similarity of results for model and field scale footings indicates that this design approach is independent of size.

STATISTICAL UPLIFT CALCULATION

Assuming that K_u and ϕ are normally distributed variables (Lumb, 1966), a normal distribution for foundation strength can be constructed by applying the Rosenblueth (1975) method to Equation 1, as described below. A statistical uplift design may be performed by comparing this foundation strength distribution to foundation load distributions to determine the probability of foundation failure. By repeating the comparison with different "trial" foundation sizes, a foundation with an acceptable probability of failure can be obtained.

The Rosenblueth method requires the mean and standard deviation values of K_u and ϕ as input. Analysis of the data showed the soil K_u values had a coefficient of variance (V_{K_u}) ranging from 19% to 39%, with 30% representing a mean value. Using this value, the coefficient of variance for ϕ (V_ϕ) of 10% reported by Harr (1977) and K_u selected from Figure 6, a normal distribution for foundation strength (Q_u) can be determined from Equations 2 to 7.

$$\bar{Q}_u = 1/4 (Q_u^{++} + Q_u^{+-} + Q_u^{-+} + Q_u^{--}) \dots\dots\dots(2)$$

$$\text{and } \sigma_{Qu} = (1/4[(Q_u^{++})^2 + (Q_u^{+-})^2 + (Q_u^{-+})^2 + (Q_u^{--})^2] - (Q_u)^2)^{1/2} \dots\dots\dots(3)$$

where $\bar{Q}_u$ is the mean ultimate uplift capacity

σ_{Qu} is the standard deviation of ultimate uplift capacity

$$Q_u^{++} = 1/2 \gamma BD^2 (\bar{K}_u + \sigma_{Ku}) \tan (\bar{\phi} + \sigma_{\phi}) + W \dots\dots\dots(4)$$

$$Q_u^{+-} = 1/2 \gamma BD^2 (\bar{K}_u + \sigma_{Ku}) \tan (\bar{\phi} - \sigma_{\phi}) + W \dots\dots\dots(5)$$

$$Q_u^{-+} = 1/2 \gamma BD^2 (\bar{K}_u - \sigma_{Ku}) \tan (\bar{\phi} + \sigma_{\phi}) + W \dots\dots\dots(6)$$

$$Q_u^{--} = 1/2 \gamma BD^2 (\bar{K}_u - \sigma_{Ku}) \tan (\bar{\phi} - \sigma_{\phi}) + W \dots\dots\dots(7)$$

where $\bar{K}_u$ is the mean value of the uplift coefficient from Figure 4

σ_{Ku} is the standard deviation of the uplift coefficient from Equation 8

$\bar{\phi}$ is the mean angle of internal friction determined by field tests

and σ_{ϕ} is the standard deviation of the angle of internal friction from Equation 9

$$\sigma_{Ku} = V_{Ku} \times \bar{K}_u = 0.30 \bar{K}_u \dots\dots\dots(8)$$

$$\sigma_{\phi} = V_{\phi} \times \bar{\phi} = 0.10 \bar{\phi} \dots\dots\dots(9)$$

The probability of foundation failure may be calculated by comparing the above normal distribution for foundation strength to a single deterministic value for foundation load. The foundation load value should not include any factored loads or safety factors. For a more realistic determination of the probability of foundation failure, the foundation strength distribution should be compared to load distributions derived from the extreme value distributions for ice and wind near the tower line (Krishnasamy and Tabatabei, 1988). As either wind or wind-on-ice may be critical for foundation design, both these load distributions must be compared to the selected foundation strength distribution. A Monte Carlo procedure is used to calculate the probability of failure for each pair of load and strength distributions. The process must be repeated for each foundation size until an acceptable probability of failure is achieved, as illustrated below.

Typical Example

To demonstrate the use of this method, the probability of failure is calculated for typical Ontario Hydro 500 kV tower line foundations located in dense sand near Toronto. Using Toronto weather data and the following typical tower line parameters,

conductor diameter = 24 mm
span between tower = 267 m
average conductor height = 27 m above ground at tower
base width of tower = 7.6 m
4 conductors per bundle
6 bundles of conductors

two extreme value distributions of loading were calculated for the tower line. For wind speed, the location parameter was 74 km/h and the scale parameter was 5.32. For combined wind and ice loading, the location parameter was 1488 N and the scale parameter was 130.7.

Existing deterministic design methods would typically require augered footings having a diameter of 1.5 m and a depth of 6 m to resist these loads. The capacity of this size of footing plus that of two shorter length footings was determined from Equations 1 to 3 by substituting the foundation dimensions shown in Table XIII and the following parameters:

$$\phi = 39^{\circ}$$

$$\sigma_{\phi} = 3.9^{\circ}$$

$$\gamma = 1763 \text{ kg/m}$$

$$\sigma_{Ku} = 0.64 \text{ (from Table XII)}$$

and K_u from Figure 6 using the appropriate D/B ratio for each footing. The mean and standard deviation parameters of the normal distribution resulting from these calculations are summarized in Table XIII. For convenience of calculation of probability of failure, the foundation compression capacity has been assumed to be equal or greater than the uplift capacity.

TABLE XIII

Foundation Capacity in Dense Sand

Foundation Size		Uplift Capacity Distribution Parameters	
<u>Diameter (m)</u>	<u>Length (m)</u>	<u>Mean (kN)</u>	<u>Standard Deviation (kN)</u>
1.5	6.1	3575	910
1.5	5.2	2452	662
1.5	4.3	1584	442

Convoluting these load and capacity distributions, using the procedure described above, yields probability of failure values ranging from 5×10^{-5} to 2.73×10^{-4} as shown in Table XIV.

TABLE XIV

Probability of Failure of Tower Foundations in Dense Sand

Foundation Size		Probability of Failure	
Diameter (m)	Length (m)	Wind Loading	Wind & Ice Loading
1.5	6.1	0.000 057	0.000 050
1.5	5.2	0.000 163	0.000 153
1.5	4.3	0.000 273	0.000 233

ANALYSIS AND DISCUSSION

The foregoing probabilistic analysis shows that a typical tower foundation in sand near Toronto has a probability of failure of approximately 5×10^{-5} . By reducing the foundation length 30%, from 6.1 m to 4.3 m the probability of failure is increased some 460% to 2.3×10^{-4} . By comparison, Whitman (1981) indicated that an annual probability of failure between 1×10^{-3} and 1×10^{-2} was accepted for foundations. Meyerhof (1970) suggested that probability of failure values between 1×10^{-4} and 1×10^{-3} should be equivalent to foundations designed with factor of safety of 3, where such foundations historically had an acceptable failure rate. These results indicate that the typical foundation design could support towers spaced at a greater span length. Alternatively, augered footings up to about 30% shorter may be considered as these foundations have a probability of failure nearer to the range of published acceptable values. Either approach would yield considerable construction cost savings.

Many researchers are now finding the beta distribution provides a better model for soil parameters than the normal distribution (Harr, 1977; Harrop-Williams, 1986; Lumb, 1970). The fact that uplift resistance can never be less than the weight of the footing plus a lower bound value of soil friction (Poorooshasb, 1980) also indicates that a skewed type distribution would be more appropriate for K_u . Further work is required in this area. This improvement in methodology would certainly reduce the probability of failure values shown in the example. However, a more rigorous accounting of the forces acting on the foundations would include the forces acting on the skywires and on the tower structure which were neglected in the example. It is anticipated that these would offset the above reduction by increasing the probability of foundation failures.

SUMMARY

Back analysis of a large number of uplift load tests in cohesionless soils was performed using the K_u theory. The lateral earth pressure coefficients (K_u) obtained range from near K_0 , for loose soils, to near K_p , for dense soils. A

wide range of footing geometries was studied permitting a more complete comparison of actual tests versus theory than had been previously reported. After statistical analysis, the test data showed reasonable agreement with the theory for dense soils but this agreement deteriorated as soils became progressively looser. The data did provide strong support for the bilinear shape originally proposed for the K_u versus D/B relationship and the basic premises underlying this relationship.

A design chart for selecting K_u values was constructed. Measures of scatter of the K_u and ϕ values in the various density bands were calculated using statistics. As the statistics were performed on a range of ϕ values, the analysis is believed to have overestimated the true scatter of K_u . A method for calculating uplift resistance using probability theory was also presented.

Further work in selecting calculations for typical tower foundations in sand, subjected to Toronto weather loads, indicate that significant reductions in foundation size may be achieved using probability-based design techniques. Refinement of the input parameters and selection of an appropriate level of probability of failure is required to transfer this technique to production foundation design.

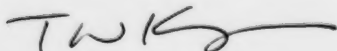
Results from full scale and model tests yielded similar values for K_u , indicating that this design approach is independent of scale effects. This suggests that the K_u method may be applied to a wide range of uplifted structures ranging from tie backs to soil anchors to large caissons.

The study also pointed out the need for better documentation of soil properties in load tests and for more uplift load tests on slender foundations in dense to very dense soils.

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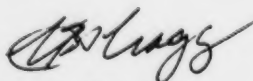
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TABLE I

Ku vs D/B Calculated from Field Ultimate Loads for Cylindrical Caissons in Dense Sands ($\phi \geq 36^\circ$)

Column	Data Point Number (1)	D/B Ratio (2)	Calculated Ku (3)	Case Study Reference Number (4)	Referenced Author (5)	Internal Friction Angle ϕ° (6)	ϕ° Determined By: (7)	Certainty of Data (8)
	1	3.33	2.17	11B #1	EPRI, 1982	37	(vi)	A
	2	3.81	1.34	18A #1	Stern et al, 1976	36.5	(i) Average N-count ~30	C
	3	3.74	1.79	18A #2	Stern et al, 1976	36.5	(i) Average N-count ~30	C
	4	4.00	3.48	11L #1	EPRI, 1982	40	(vii)	A
	5	4.00	5.99	11N #1	EPRI, 1982	37	(i) Average N-count ~36	C
	6	1.80	3.53	11"0" #1	EPRI, 1982	40	(vii)	A
	7	2.08	3.06	11"0" #2	EPRI, 1982	40	(vii)	A
	8	2.68	2.81	11"0" #3	EPRI, 1982	40	(vii)	A
	9	1.94	1.47	8A #1	Adams & Hayes, 1967	40	(v) Given assumed value	A
	10	2.33	1.80	8A #2	Adams & Hayes, 1967	40	(v) Given assumed value	C
	11	3.81	2.14	8A #3	Adams & Hayes, 1967	40	(v) Given assumed value	C
	12	1.95	1.59	5A #30a	Ontario Hydro Report No 64-317, 1964	40	(iv) Given value	C
	13	2.73	2.36	5A #30b	Ontario Hydro, 1964	40	(iv) Given value	C
	14	3.76	2.24	5A #30c	Ontario Hydro, 1964	40	(iv) Given value	C
	15	3.88	2.95	5A #30d	Ontario Hydro, 1964	40	(iv) Given value	C
	16	3.76	1.47	5A #30e	Ontario Hydro, 1964	40	(iv) Given value	C
	17	3.03	6.62	6A #1	Ontario Hydro Report No 65-145 (1965)	36	(vii)	A
	18	5.60	2.70	22A #1	Cragg, 1985; Essa	36	(i) Average N-count ~30	C
	19	5.70	2.64	22A #2	Cragg, 1985; Essa	36	(i) Average N-count ~30	C
	20	5.00	3.14	22A #3	Cragg, 1985; Essa	36	(i) Average N-count ~30	C
	21	4.70	2.55	22A #4	Cragg, 1985; Essa	36	(i) Average N-count ~30	C
	22	4.10	2.25	22A #5	Cragg, 1985; Essa	36	(i) Average N-count ~30	C
	23	4.10	2.14	22A #6	Cragg, 1985; Essa	36	(i) Average N-count ~30	C
	24	3.00	2.33	22A #7	Cragg, 1985; Essa	36	(i) Average N-count ~30	C
	25	3.00	1.52	22A #8	Cragg, 1985; Essa	36	(i) Average N-count ~30	C
	26	7.10	3.06	22A #9a	Cragg, 1985; Essa	36	(i) Average N-count ~30	C
	27	7.10	2.94	22A #9b	Cragg, 1985; Essa	36	(i) Average N-count ~30	C
	28	6.10	3.67	22A #10	Cragg, 1985; Essa	36	(i) Average N-count ~30	C
	29	5.00	2.84	22A #11	Cragg, 1985; Essa	36	(i) Average N-count ~30	C
	30	3.80	1.79	22A #12	Cragg, 1985; Essa	36	(i) Average N-count ~30	C

NOTE: Data Point #9 could be same as #12
 Data Point #10 could be same as #13
 Data Point #11 could be same as #14

* shown in Appendix as Case 111

General Notes to Tables I to VIII follow Table VIII

TABLE II

Ku vs D/B Calculated from Field Ultimate Loads for Cylindrical Caissons in Compact Sands ($33^\circ \leq \phi \leq 36^\circ$)

Data Point Number	D/B Ratio	Calculated Ku	Case Study Reference Number	Referenced Author	Internal Friction Angle ϕ°	ϕ° Determined By:	Certainty of Data
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	5.69	1.97	3B #13	Downs & Chieurrzzi, 1966	35	(vi)	A
2	6.20	1.82	3B #14	Downs & Chieurrzzi, 1966	35	(vi)	A
3	4.15	2.28	11E #1	EPRI, 1982	34	(i)	C
4	4.09	1.88	11J #1	EPRI, 1982	33.5	(i)	C
5	8.25	1.22	14A #2	Paterson & Urie, 1964 & EPRI	35	(v)	C
6	9.01	1.25	14A #2	Paterson & Urie, 1964 & EPRI	35	(v)	C
7	9.75	1.34	14A #3	Paterson & Urie, 1964 & EPRI	35	(v)	C
8	10.51	0.82	14A #4	Paterson & Urie, 1964 & EPRI	35	(v)	C
9	8.25	1.66	14A #5	Paterson & Urie, 1964 & EPRI	35	(v)	C
10	8.25	1.44	14A #6	Paterson & Urie, 1964 & EPRI	35	(v)	C
11	6.25	1.47	3A #6c	Downs & Chieurrzzi, 1966	34	(vi)	C
12	9.40	1.27	3A #7c	Downs & Chieurrzzi, 1966	34	(vi)	A
13	9.57	1.42	3A #8	Downs & Chieurrzzi, 1966	34	(vi)	A
14	6.25	1.52	3B #10c	Downs & Chieurrzzi, 1966	35	(vi)	A
15	6.63	0.71	3B #11c	Downs & Chieurrzzi, 1966	34	(vi)	A
16	5.69	1.96	3B #13	Downs & Chieurrzzi, 1966	35	(vi)	A
17	6.00	2.34	22B #1	Cragg, 1985; Nanticoke	35	(i)	A
18	5.20	3.89	22B #2	Cragg, 1985; Nanticoke	35	(i)	C
19	4.00	1.67	22B #3	Cragg, 1985; Nanticoke	35	(i)	C
20	3.20	1.52	22B #4	Cragg, 1985; Nanticoke	35	(i)	C

Note: Paterson & Urie tests shown in Appendix as Case 11M

General Notes to Tables I to VIII follow Table VIII

TABLE VII

Ku vs D/B Calculated from Field Ultimate Loads for Cylindrical Caissons in Compact Sands ($30^\circ \leq \phi \leq 33^\circ$)

Data Point Number	D/B Ratio (2)	Calculated Ku (3)	Case Study Reference Number (4)	Referenced Author (5)	Internal Friction Angle ϕ° (6)	ϕ° Determined By: (7)	Certainty of Data (8)
1	3.33	4.30	11A #1	EPRI, 1982	30	(i) Average N-count ~10	C
2	15.50	2.13	11D #1	EPRI, 1982	32.5	(i) Average N-count ~18	C
3	6.86	2.55	11F #1	EPRI, 1982	32.5	(i) Average N-count ~18	C
4	10.28	3.48	11G #1	EPRI, 1982	32.5	(i) Average N-count ~13	C
5	5.90	0.99	11H #1	EPRI, 1982	31.2	(i) Average N-count ~17	C
6	6.00	0.99	11H #2	EPRI, 1982	32	(i) Average N-count ~17	C
7	3.17	1.37	7A #1	Ontario Hydro Report No 65-195, 1965	32	(v) Given assumed value	C
8	3.33	0.98	7A #2	Ontario Hydro, 1965	32	(v) Given assumed value	C
9	2.99	1.51	7A #3	Ontario Hydro, 1965	32	(v) Given assumed value	C
10	3.33	0.28	7B #1	Ontario Hydro, 1965	32	(v) Given assumed value	C
11	22.85	0.34	17A #1	Sowa, 1970	31.5	(i) Average N-count ~13	C
12	32.10	0.56	17A #2	Sowa, 1970	31.5	(i) Average N-count ~13	C
13	7.00	1.81	10A #1	Adams & Hayes, 1967	33	(i) Average N-count ~20	C
14	7.00	1.81	10A #2	Adams & Hayes, 1967	33	(i) Average N-count ~20	C
15	7.00	2.04	10A #3	Adams & Hayes, 1967	34	(i) Average N-count ~22	C
16	3.72	0.60	11C #1	EPRI, 1982	31	(i) Average N-count ~12	C
17	4.75	0.61	11C #2	EPRI, 1982	31	(i) Average N-count ~12	C
18	10.49	3.43	11K #1	EPRI, 1982	30	(vii) Given assumed value	A
19	10.49	2.99	11K #2	EPRI, 1982	30	(vii) Given assumed value	A
20	8.00	3.14	8B #1	Adams & Hayes, 1967	30	(v) Given assumed value	C
21	2.38	1.03	8D #1	Adams & Hayes, 1967	30	(v) Given assumed value	C
22	3.60	1.10	8E #1	Adams & Hayes, 1967	30	(iv) Given value	C
23	2.80	1.70	8E #2	Adams & Hayes, 1967	30	(iv) Given value	C
24	2.33	1.54	8E #3	Adams & Hayes, 1967	30	(iv) Given value	C
25	10.75	4.04	13A #24a	Mors, 1959	30	(vii) Given value	A
26	10.75	4.29	13B #24b	Mors, 1959	30	(vii) Given value	A

NOTE: Data point #5,6 same as #13,14,15

General Notes to Tables I to VIII follow Table VIII

TABLE IV

K_u vs D/B Calculated from Field Ultimate Loads for Cylindrical Caissons in Loose Sands ($\phi \leq 30^\circ$)

Data Point Number	D/B Ratio	Calculated K_u	Case Study Reference Number	Referenced Author	Internal Friction Angle ϕ°	ϕ° Determined By:	Certainty of Data (8)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	6.63	1.82	3A #1	Downs & Chieurrzzi, 1966	24.7	(vi)	A
2	6.62	3.16	3A #2	Downs & Chieurrzzi, 1966	24.7	(vi)	A
3	6.25	1.02	3A #6a	Downs & Chieurrzzi, 1966	24.7	(vi)	A
4	6.63	0.79	3B #11a	Downs & Chieurrzzi, 1966	28	(vi) Given in situ $\phi_0 = 24.7^\circ$	A
5	6.63	0.99	3B #12	Downs & Chieurrzzi, 1966	28	(vi) Given in situ $\phi_0 = 35^\circ$	A
6	2.81	2.77	4A #1	Giffels et al, 1960	28	(vi) Given in situ $\phi_0 = 35^\circ$	A
7	3.61	1.52	4B #1	Giffels et al, 1960	≥ 28	(i) Average N-count ~ 2	A
8	2.81	2.34	4B #2	Giffels et al, 1960	≥ 28	(i) Average N-count ~ 4	A
9	2.99	1.63	4B #3	Giffels et al, 1960	≥ 28	(i) Average N-count ~ 4	A
10	2.35	2.11	4B #4	Giffels et al, 1960	≥ 28	(i) Average N-count ~ 4	A
11	4.00	1.15	4B #5	Giffels et al, 1960	≥ 28	(i) Average N-count ~ 4	A
12	4.35	1.10	4B #6	Giffels et al, 1960	≥ 28	(i) Average N-count ~ 4	A
13	6.25	0.53	3B #10a	Downs & Chieurrzzi, 1966	28	(i) Average N-count ~ 4	A
14	6.25	1.25	3A #6b	Downs & Chieurrzzi, 1966	30	(vi) Given in situ $\phi_0 = 35^\circ$	A
15	9.40	1.10	3A #7b	Downs & Chieurrzzi, 1966	30	(vi)	A
16	6.37	1.00	3B #9	Downs & Chieurrzzi, 1966	30	(vi)	A
17	6.25	0.61	3B #10b	Downs & Chieurrzzi, 1966	30	(vi)	A
18	6.25	0.33	3B #11b	Downs & Chieurrzzi, 1966	30	(vi)	A

TABLE V

Ku vs D/B Calculated from Laboratory Ultimate Loads for Cylindrical Caissons in Dense Sands ($\phi \geq 36^\circ$)

Data Point Number	D/B Ratio	Calculated Ku	Case Study Reference Number	Referenced Author	Internal Friction Angle ϕ°	ϕ° Determined By:	Certainty of Data
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	5.00	2.87	10B #2	Ontario Hydro Report No 77-538-H Ismael, 1977	39.5	(ii) Direct shear	C
2	5.00	2.87	10B #3	Ontario Hydro Report No 77-538-H Ismael, 1977	39.5	(ii) Direct shear	C
3	3.00	2.99	16B #1	Esquivel-Piaz, 1967 (in Seeley et al 1975)	43	(iv) Given value	C
4	4.50	3.48	16B #2	Esquivel-Piaz, 1967	43	(iv) Given value	C
5	10.10	4.20	16B #3	Esquivel-Piaz, 1967	43	(iv) Given value	C
6	10.00	2.19	19A #2	Withiam & Kulhawy, 1978	47	(iv) Given value	C
7	10.00	3.32	19A #3	Withiam & Kulhawy, 1978	47	(iv) Given value	C
8	10.00	2.66	20B #6	Withiam & Kulhawy, 1978	38	(iv) Given value	C
9	4.00	5.05	2C #1	Das, 1983	40.5	(iv) Given value	C
10	4.50	4.66	2C #2	Das, 1983	40.5	(iv) Given value	C
11	8.00	2.95	2C #3	Das, 1983	40.5	(iv) Given value	C
12	10.00	3.23	2C #4	Das, 1983	40.5	(iv) Given value	C
13	12.00	3.37	2C #5	Das, 1983	40.5	(iv) Given value	C
14	14.00	3.71	2C #6	Das, 1983	40.5	(iv) Given value	C
15	16.00	3.47	2C #7	Das, 1983	40.5	(iv) Given value	C
16	18.00	3.49	2C #8	Das, 1983	40.5	(iv) Given value	C
17	20.00	3.44	2C #9	Das, 1983	40.5	(iv) Given value	C
18	22.00	3.29	2C #10	Das, 1983	40.5	(iv) Given value	C
19	24.00	3.09	2C #11	Das, 1983	40.5	(iv) Given value	C
20	3.00	1.80	22C #1	Cragg & Tsang, 1985	38	(ii) Direct shear	C
21	4.00	3.19	22C #2	Cragg & Tsang, 1985	38	(ii) Direct shear	C
22	5.00	2.98	22C #3	Cragg & Tsang, 1985	38	(ii) Direct shear	C
23	6.00	3.39	22C #4	Cragg & Tsang, 1985	38	(ii) Direct shear	C
24	3.00	3.02	22C #5	Cragg & Tsang, 1985	38	(ii) Direct shear	C
25	4.00	2.09	22C #6	Cragg & Tsang, 1985	38	(ii) Direct shear	C

General Notes to Tables I to VIII follow Table VIII

TABLE V (cont'd)

Data Point Number	D/B Ratio	Calculated Ku	Case Study Reference Number	Referenced Author	Internal Friction Angle ϕ	ϕ Determined By:	Certainty of Data
26	5.00	2.55	22C #7	Cragg & Tsang, 1985	38	(ii) Direct shear	C
27	6.00	3.14	22C #8	Cragg & Tsang, 1985	38	(ii) Direct shear	C
28	3.00	2.46	22C #9	Cragg & Tsang, 1985	38	(ii) Direct shear	C
29	4.00	2.37	22C #10	Cragg & Tsang, 1985	38	(ii) Direct shear	C
30	5.00	2.84	22C #11	Cragg & Tsang, 1985	38	(ii) Direct shear	C
31	6.00	3.42	22C #12	Cragg & Tsang, 1985	38	(ii) Direct shear	C
32	3.00	2.53	22C #13	Cragg & Tsang, 1985	38	(ii) Direct shear	C
33	4.00	2.40	22C #14	Cragg & Tsang, 1985	38	(ii) Direct shear	C
34	5.00	2.92	22C #15	Cragg & Tsang, 1985	38	(ii) Direct shear	C
35	6.00	2.85	22C #16	Cragg & Tsang, 1985	38	(ii) Direct shear	C
36	4.50	3.83	22C #17	Cragg & Tsang, 1985	38	(ii) Direct shear	C
37	6.00	3.42	22C #18	Cragg & Tsang, 1985	38	(ii) Direct shear	C
38	7.50	3.52	22C #19	Cragg & Tsang, 1985	38	(ii) Direct shear	C
39	9.00	4.75	22C #20	Cragg & Tsang, 1985	38	(ii) Direct shear	C
40	4.50	2.82	22C #21	Cragg & Tsang, 1985	38	(ii) Direct shear	C
41	6.00	2.58	22C #22	Cragg & Tsang, 1985	38	(ii) Direct shear	C
42	7.50	3.64	22C #23	Cragg & Tsang, 1985	38	(ii) Direct shear	C
43	9.00	4.98	22C #24	Cragg & Tsang, 1985	38	(ii) Direct shear	C

General Notes to Tables I to VIII follow Table VIII

TABLE VI

Ku vs D/B Calculated from Laboratory Ultimate Loads for Cylindrical Caissons in Compact Sands ($33^\circ \leq \phi \leq 36^\circ$)

Data Point Number	D/B Ratio	Calculated Ku	Case Study Reference Number	Referenced Author	Internal Friction Angle ϕ°	ϕ° Determined By:	Certainty of Data
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	10.00	1.99	20B #2	Withiam & Kulhawy, 1978	34	(v) Given assumed value	C
2	10.00	2.77	20B #5	Withiam & Kulhawy, 1978	35	(v) Given assumed value	C
3	10.00	2.00	20A #1	Withiam & Kulhawy, 1978	34	(v) Given assumed value	C
4	5.00	1.33	20A #2	Withiam & Kulhawy, 1978	34	(v) Given assumed value	C
5	20.00	1.80	20A #3	Withiam & Kulhawy, 1978	34	(v) Given assumed value	C
6	10.00	1.20	20A #4	Withiam & Kulhawy, 1978	34	(v) Given assumed value	C
7	10.00	2.12	20B #3	Withiam & Kulhawy, 1978	36	(v) Given assumed value	C
8	4.00	2.28	2B #1	Das, 1983	34	(iv) Given value	C
9	6.00	2.23	2B #2	Das, 1983	34	(iv) Given value	C
10	8.00	1.82	2B #3	Das, 1983	34	(iv) Given value	C
11	10.00	1.84	2B #4	Das, 1983	34	(iv) Given value	C
12	12.00	1.65	2B #5	Das, 1983	34	(iv) Given value	C
13	14.00	1.68	2B #6	Das, 1983	34	(iv) Given value	C
14	16.00	1.64	2B #7	Das, 1983	34	(iv) Given value	C
15	18.00	1.41	2B #8	Das, 1983	34	(iv) Given value	C
16	20.00	1.32	2B #9	Das, 1983	34	(iv) Given value	C
17	22.00	1.24	2B #10	Das, 1983	34	(iv) Given value	C
18	24.00	1.14	2B #11	Das, 1983	34	(iv) Given value	C

General Notes to Tables I to VIII follow Table VIII

TABLE VII

Ku vs D/B Calculated from Laboratory Ultimate Loads for Cylindrical Caissons in Compact Sands ($30^\circ \leq \phi \leq 33^\circ$)

Data Point Number (1)	D/B Ratio (2)	Calculated Ku (3)	Case Study Reference Number (4)	Referenced Author (5)	Internal Friction Angle ϕ° (6)	ϕ° Determined By: (7)	Certainty of Data (8)
1	10.00	2.51	20B #1	Withiam & Kulhawy, 1978	32	(v) Given assumed value	C
2	10.00	1.74	20B #4	Withiam & Kulhawy, 1978	32	(v) Given assumed value	C

General Notes to Tables I to VIII follow Table VIII

TABLE VIII

Ku vs D/B Calculated from Laboratory Ultimate Loads for Cylindrical Caissons in Loose Sand ($\phi \leq 30^\circ$)

Data Point Number	D/B Ratio	Calculated Ku	Case Study Reference Number	Referenced Author	Internal Friction Angle ϕ°	ϕ° Determined By:	Certainty of Data
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	5.03	0.45	10B #1	Ontario Hydro Report No 77-558-H	30	Direct shear	C
2	4.00	0.85	16A #1	Ismael, 1977	31	(iv) Reported as loose sand	C
3	6.50	0.81	16A #2	Seeley, 1975	31	(iv) Reported as loose sand	C
4	8.00	0.90	16A #3	Seeley, 1975 & Braja, 1975	31	(iv) Reported as loose sand	C
5	10.00	0.87	16A #4	Seeley, 1975 & Braja, 1975	31	(iv) Reported as loose sand	C
6	12.00	0.78	16A #5	Seeley, 1975 & Braja, 1975	31	(iv) Reported as loose sand	C
7	14.00	0.67	16A #6	Seeley, 1975 & Braja, 1975	31	(iv) Reported as loose sand	C
8	16.00	0.64	16A #7	Seeley, 1975 & Braja, 1975	31	(iv) Reported as loose sand	C
9	18.00	0.61	16A #8	Seeley, 1975 & Braja, 1975	31	(iv) Reported as loose sand	C
10	20.00	0.61	16A #9	Seeley, 1975 & Braja, 1975	31	(iv) Reported as loose sand	C
11	22.00	0.56	16A #10	Seeley, 1975 & Braja, 1975	31	(iv) Reported as loose sand	C
12	24.00	0.53	16A #11	Seeley, 1975 & Braja, 1975	31	(iv) Reported as loose sand	C
13	1.50	0.46	16B #1	Esquivel-Diaz 1967 (in Seeley et al 1975)	33	(iv) Reported as loose sand	C
14	3.00	0.61	16B #2	Esquivel-Diaz, 1967	33	(iv) Reported as loose sand	C
15	4.50	0.49	16B #3	Esquivel-Diaz, 1967	33	(iv) Reported as loose sand	C
16	6.00	0.57	16B #4	Esquivel-Diaz, 1967	33	(iv) Reported as loose sand	C
17	10.00	0.49	16B #5	Esquivel-Diaz, 1967	33	(iv) Reported as loose sand	C
18	10.00	0.30	19A #1	Withiam & Kulhawy, 1978	34.7	(iv) Reported as loose sand	C
19	4.00	1.36	2C #1	Das, 1983	31	(iv) Reported as loose sand	C
20	6.00	1.21	2C #2	Das, 1983	31	(iv) Reported as loose sand	C
21	8.00	1.02	2C #3	Das, 1983	31	(iv) Reported as loose sand	C
22	10.00	0.87	2C #4	Das, 1983	31	(iv) Reported as loose sand	C
23	12.00	0.76	2C #5	Das, 1983	31	(iv) Reported as loose sand	C
24	14.00	0.67	2C #6	Das, 1983	31	(iv) Reported as loose sand	C
25	16.00	0.65	2C #7	Das, 1983	31	(iv) Reported as loose sand	C
26	18.00	0.67	2C #8	Das, 1983	31	(iv) Reported as loose sand	C
27	20.00	0.66	2C #9	Das, 1983	31	(iv) Reported as loose sand	C
28	22.00	0.59	2C #10	Das, 1983	31	(iv) Reported as loose sand	C
29	24.00	0.53	2C #11	Das, 1983	31	(iv) Reported as loose sand	C

General Notes to Tables I to VIII follow Table VIII

General Notes to Tables I to VIII

Column (1) Data Point Number - refers to an individual data point supported by a pullout test and calculated D/B and Ku value.

Column (2) D/B Ratio - embedment depth (D) to diameter (B) ratio of the test footing.

Column (3) Calculated Ku - determined from the ultimate uplift load on the test footing.

Column (4) Case Study Reference Number - refers to the numbered case studies in Appendix A (eg, 18A #2 is case study 18 at site A, test footing #2).

Column (5) Referenced Author - reference for the case study, detailed reference given in Appendix A.

Column (6) Internal Friction Angle, ϕ^0 - angle of soil shearing resistance used in determining Ku.

Column (7) ϕ^0 Determined by - the method used to obtain ϕ is shown by the following code:

- (i) average N value from Standard Penetration Test (SPT)
- (ii) direct shear test - reported value
- (iii) triaxial test - reported value
- (iv) ϕ value provided in the reference but the method of obtaining ϕ was not reported
- (v) ϕ value provided in the reference was assumed (ie not based on field tests)
- (vi) no pertinent data in the reference; ϕ assumed by the present writers based on description of test footing installation in the reference
- (vii) no pertinent data in the reference; ϕ assumed by the present writers based on description of in situ soil conditions prior to footing installation in the reference.

Column (8) Certainty of Data - an indicator of the reliability of the Ku values obtained from each case study coded as follows:

"C" - certain: all the essential data needed to calculate Ku was reported in the case study (ie no assumed values required)

"A" - assumed: the case study lacked essential data so assumed values were used to calculate Ku

TABLE IX

Points Excluded from the Data Base

<u>Date Set</u>	<u>Table Number</u>	<u>Data Point Number</u>	<u>Case Study Reference</u>	<u>Reason for Exclusion</u>
Field (Dense)	I	2	18A #1	(1) soil weakened by construction method
		3	18A #2	(1) soil weakened by construction method
		5	11N #1	(3) two ⁺ soil layers with different properties
		6	11"0" #1	(6) anomalies in foundation construction or testing
		7	11"0" #2	(6) anomalies in foundation construction or testing
		16	5A #30e	(2) footing weakened by previous testing
		17	6A #1	(4,5) non-cylindrical footing/ inclined load
		18	22A #1	(6) anomalies in foundation construction or testing
		19	22A #2	(6) anomalies in foundation construction or testing
		26	22A #9a	(6) anomalies in foundation construction or testing
		27	22A #9b	(6) anomalies in foundation construction or testing
Field (Compact)	II	15	3B #11c	(1,2) soil weakened by construction method/ footing weakened by previous testing
Field (Compact)	III	1	11A #1	(3) two ⁺ soil layers with different properties
		8	7A #2	(1) soil weakened by construction method
		10	7B #1	(1) soil weakened by construction method
		11	17A #1	(9) very large D/B
		12	17A #2	(9) very large D/B
		15	10A #3	(10) duplicate reporting of test
		16	11C #1	(3) two ⁺ soil layers with different properties
		17	11C #2	(3) two ⁺ soil layers with different properties
		25	13A #24a	(10) duplicate reporting of test

<u>Date Set</u>	<u>Table Number</u>	<u>Data Point Number</u>	<u>Case Study Reference</u>	<u>Reason for Exclusion</u>
Field	IV	1	3A #1	(7) Ku questionable due to assumed $\emptyset$
		2	3A #2	(7,4) Ku questionable due to assumed $\emptyset$ / non-cylindrical footing
		6	4A #1	(8) SPT N-count appears unreliable
		7	4B #1	(8) SPT N-count appears unreliable
		8	4B #2	(8) SPT N-count appears unreliable
		9	4B #3	(8) SPT N-count appears unreliable
		10	4B #4	(8) SPT N-count appears unreliable

TABLE X

Distribution of Uplift Test Data

<u>Test Scale</u>	<u>Soil Density</u>	<u>Number of Footing Tests</u>	
		<u>Found</u>	<u>Used in Analysis</u>
Field	dense	30	22
	compact	46	35
	loose	<u>18</u>	<u>11</u>
	Sub Total	94	68
Laboratory	dense	43	43
	compact	20	20
	loose	<u>29</u>	<u>29</u>
	Sub Total	<u>92</u>	<u>92</u>
	TOTAL	<u>186</u>	<u>160</u>

TABLE XI

Contributors to Data Base of Uplift Tests in Cohesionless Soils

<u>Author(s)</u>	<u>Number of Data Points</u>	<u>% of Data Base</u>
*Adams & Hayes (1967)	11	
*Cragg (1985)	17	
*Cragg & Tsang (1985a)	24	
Das (1983)	33	
Downs & Chieurrzzi (1966)	19	
EPRI (1982)	18	
Esquivez-Diaz (1967)	7	
Giffels (1960)	7	
*Ismael (1977)	3	
Mors (1959)	2	
*Ontario Hydro (1964)	5	
*Ontario Hydro (1965)	5	
Peterson & Urie (1964)	6	
Seeley (1975); Seeley & Braja (1975)	11	
Sowa (1970)	2	
Stern et al (1976)	2	
Withiam & Kulhawy (1978)	13	
<u>Major Sources</u>		
Cragg; Cragg & Tsang	41	} 124 67
Das	33	
EPRI; Kulhawy	31	
Downs & Chieurrzzi	19	
*Ontario Hydro	65	35

TABLE XII

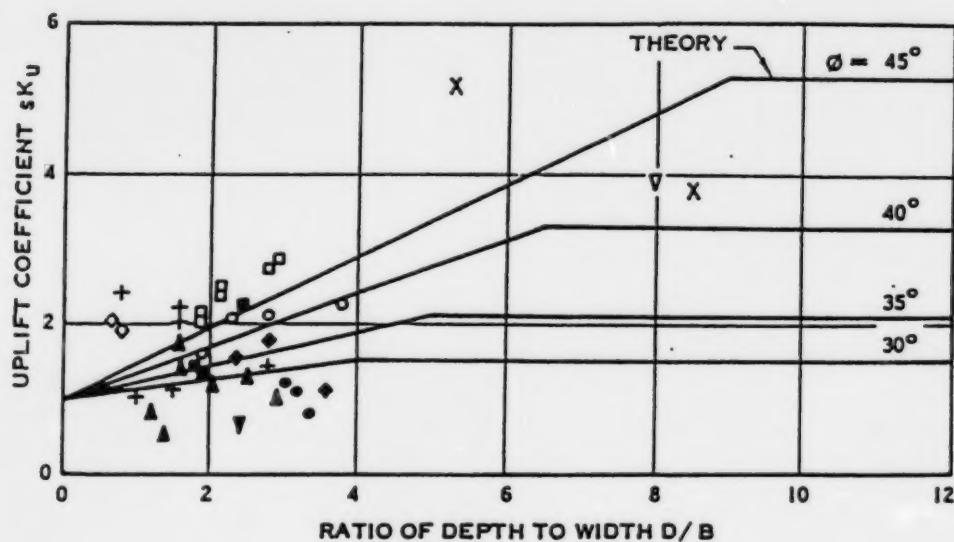
Statistical Parameters for Foundation Uplift Coefficient, K_u

Soil Density	D/B Ratio	Field Test Parameters	Model Test Parameters	Combined Data Parameters
dense	<6.5	n = 21 $S_e = \pm 0.45$ r = 69%	n = 26 $S_e = \pm 0.70$ r = 21%	n = 47 $S_e = \pm 0.64$ r = 50%
	>6.5	n = 1	n = 17 $\bar{K}_u = 3.49$ $\sigma_{K_u} = \pm 0.68$	n = 18 $\bar{K}_u = 3.46$ $\sigma_{K_u} = \pm 0.67$
compact	<5.0	n = 10 $S_e = \pm 0.31$ r = 58%	n = 2	n = 12 $S_e = \pm 0.39$ r = 37%
	>5.0	n = 25 $\bar{K}_u = 1.96$ $\sigma_{K_u} = \pm 0.89$	n = 18 $\bar{K}_u = 1.78$ $\sigma_{K_u} = \pm 0.44$	n = 43 $\bar{K}_u = 1.89$ $\sigma_{K_u} = \pm 0.74$
loose		n = 11 $\bar{K}_u = 0.90$ $\sigma_{K_u} = \pm 0.27$	n = 29 $\bar{K}_u = 0.67$ $\sigma_{K_u} = \pm 0.23$	n = 40 $\bar{K}_u = 0.75$ $\sigma_{K_u} = \pm 0.26$

n = number of data points

 $\bar{K}_u$ = average value of K_u S_e = standard error of estimate (of K_u) σ_{K_u} = standard deviation of

r = coefficient of correlation



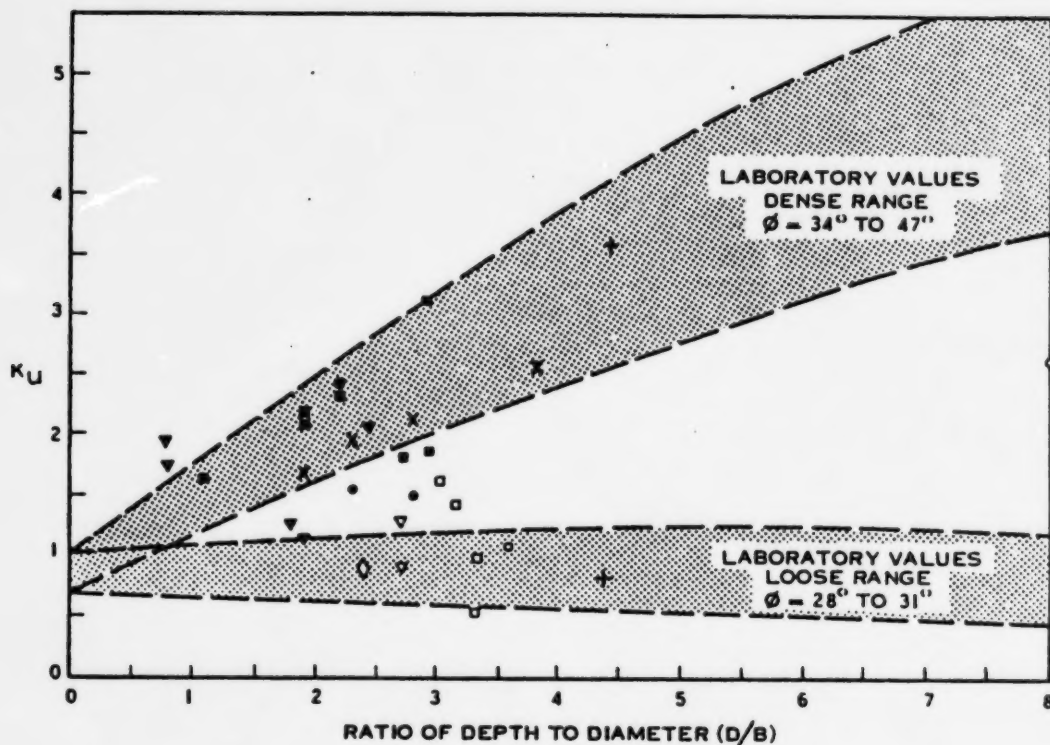
DENSE SAND

- ADAMS AND HAYES (1967) - HAMILTON $\phi = 40^\circ$
- ▽ ADAMS AND HAYES (1967) - SUDBURY $\phi = 30^\circ$
- SUTHERLAND (1965) $\phi = 36^\circ - 42^\circ$
- ◇ BALLA (1961) $\phi = 36^\circ$
- X BAKER AND KONDER (1966) $\phi = 37^\circ$
- + MORS (1959) $\phi = 35^\circ - 40^\circ$

LOOSE SAND

- ADAMS AND HAYES (1967) - ESSA $\phi = 32^\circ$
- ▽ ADAMS AND HAYES (1967) - ISLINGTON $\phi = 30^\circ$
- BALLA (1961) $\phi = 30^\circ$
- ◆ GIFFELS ET AL (1960) $\phi = 30^\circ$
- ▲ MORS (1959) $\phi = 26^\circ - 30^\circ$

FIGURE 1
COMPARISON OF K_u THEORY AND FULL SCALE TESTS
IN SAND (AFTER MEYERHOF AND ADAMS, 1968)



FULL-SCALE TESTS BY:

ONTARIO HYDRO:

- - SUDBURY (HANMER)
- x - HAMILTON
- - ESSA (EHV LINE)
- ◊ - ISLINGTON

OTHERS:

- ▼ - BALLA - LOOSE AND DENSE SAND (REF 1)
- ▼ - ELECTRICAL RES. ASSOC. - LOOSE BACKFILL (REF 4)
- - SUTHERLAND - DENSE SAND (REF 5)
- - OHIO EDISON - LOOSE SAND (REF 6)

FIGURE 2

COMPARISON OF UPLIFT COEFFICIENTS FOR FULL SCALE AND
LABORATORY TESTS (AFTER ADAMS AND RADHAKRISHNA, 1977)

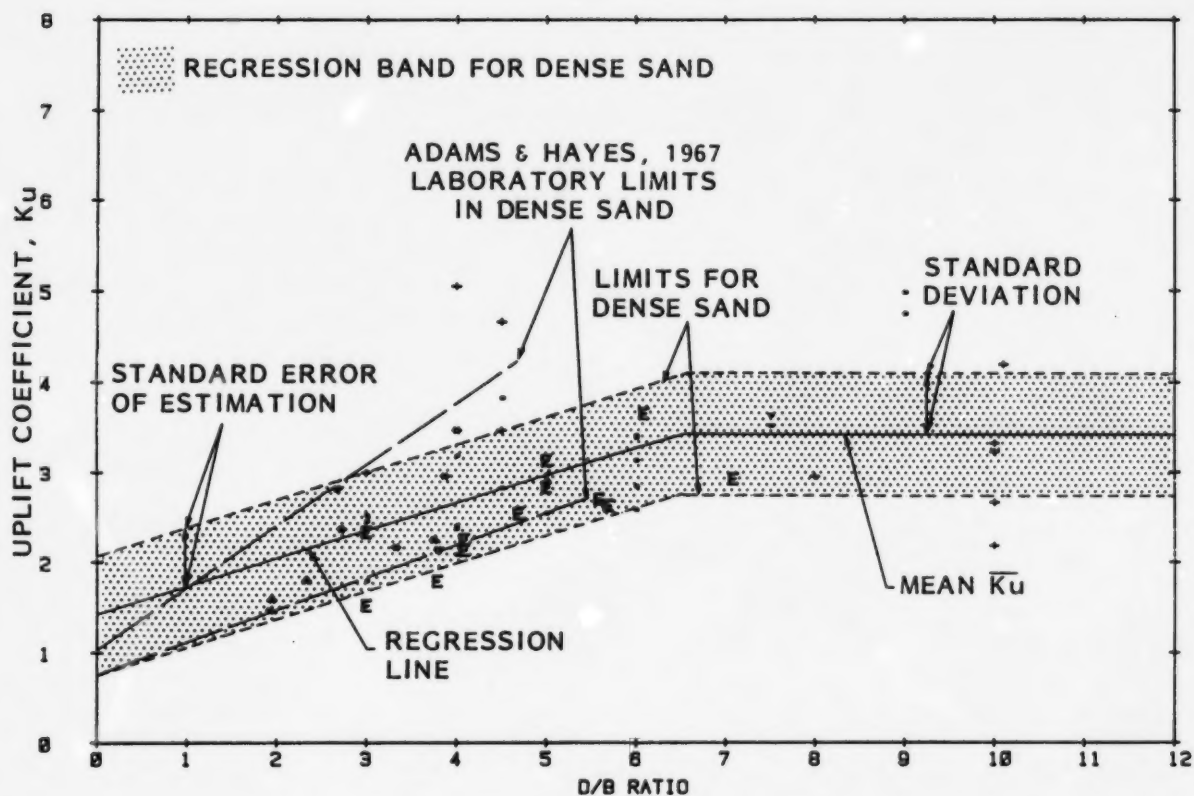


FIGURE 3
STATISTICAL ANALYSIS OF UPLIFT
COEFFICIENTS FOR DENSE SAND

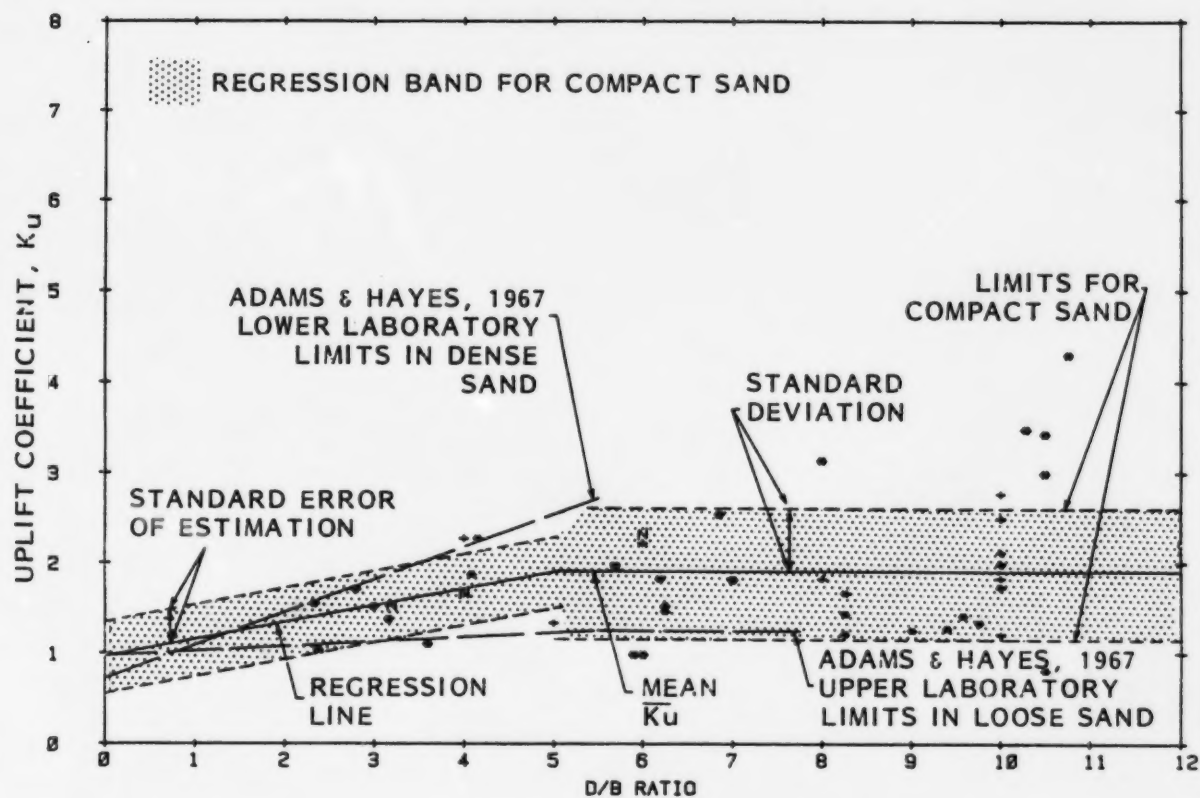


FIGURE 4
STATISTICAL ANALYSIS OF UPLIFT
COEFFICIENTS FOR COMPACT SAND

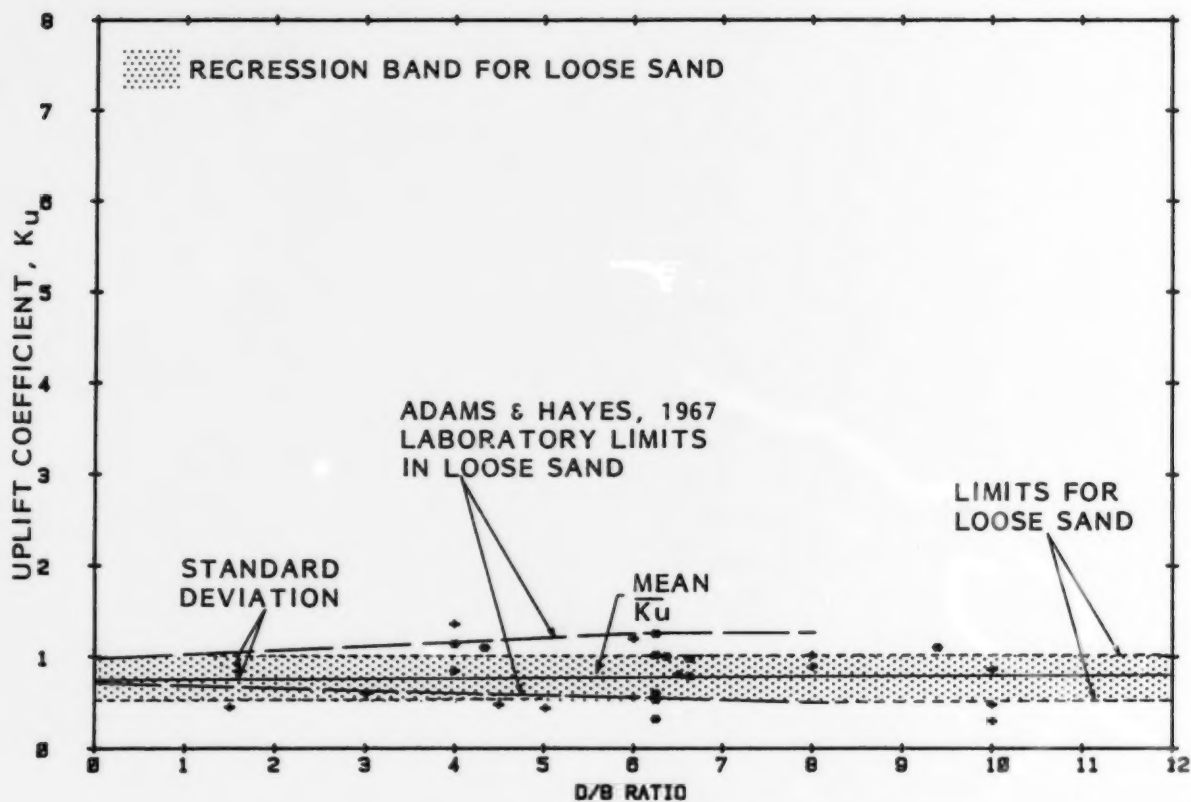


FIGURE 5
STATISTICAL ANALYSIS OF UPLIFT
COEFFICIENTS FOR LOOSE SAND

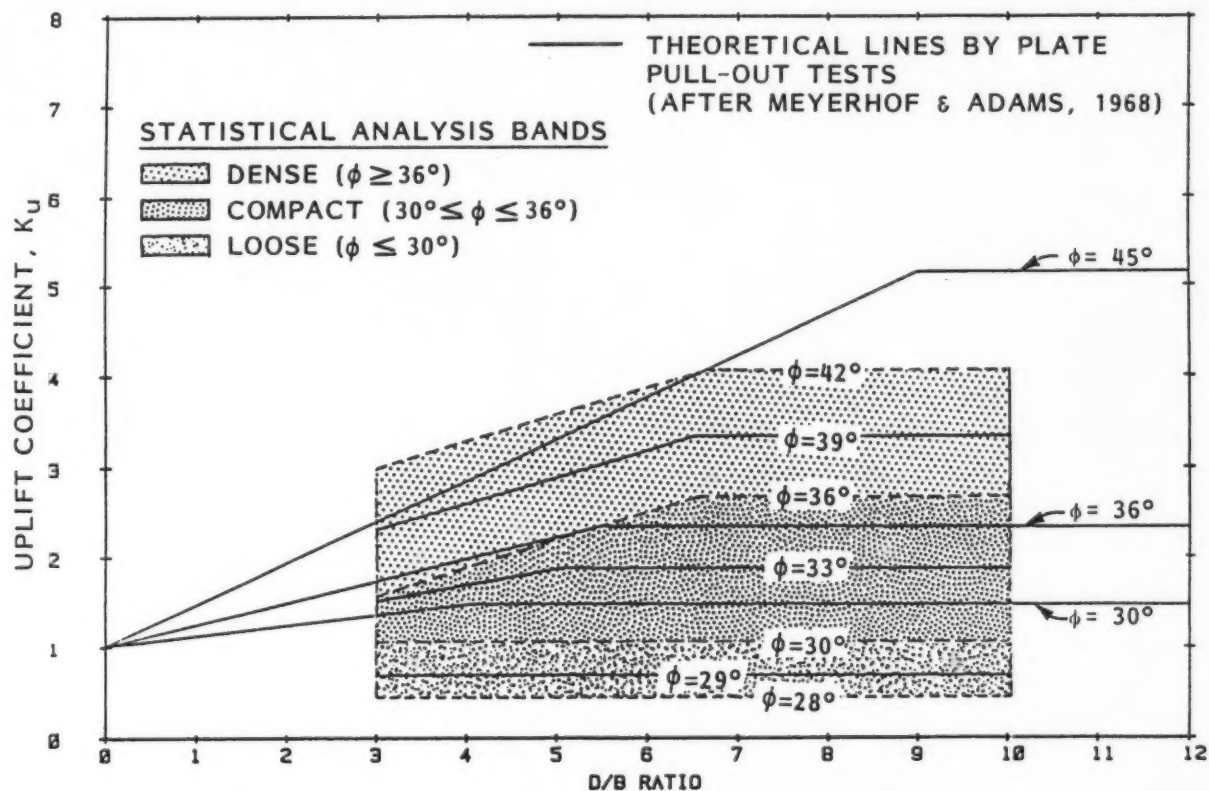


FIGURE 6
COMPARISON OF COMBINED STATISTICAL UPLIFT
COEFFICIENT BANDS WITH K_u THEORY